SUBSURFACE INVESTIGATION

PROPOSED DRAINAGE TUNNEL AND NEW BOX CULVERTS

PEARL CITY, OAHU, HAWAII

FOR

CITY AND COUNTY OF HONOLULU

DAMES & MOORE NO. 4402-037-11

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CABLE: DAMEMORE TELEX: 63-4100

September 2, 1970

Yasuo Arakaki, Consulting Engineer
914 Ala Moana Boulevard
Honolulu, Hawaii

Gentlemen:

Seven copies of our report entitled "Subsurface Investigation, Proposed Drainage Tunnel and New Box Culverts, Pearl City, Oahu, Hawaii, for City and County of Honolulu" are herewith submitted. The body of the report is divided into two sections. The first section pertains to the proposed tunnel and the second to the new box culverts. At the beginning of each section, there is a brief summary of our conclusions and recommendations.

Rock core recovered during drilling is available for viewing at our office in Honolulu and will be retained for a period of one year from the date of this report. If desired, the core could be delivered to your or your clients for safekeeping in case prospective bidders wish to examine it.

The scope of our work has conformed to that presented in our revised proposal of June 12, 1970. If we can be of further assistance to you in connection with this project, please call on us.

Yours very truly,

DAMES & MOORE

David C. Liu

David C. Liu

DCL DRR jms

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PROPOSED DRAINAGE TUNNEL
AND NEW BOX CULVERTS
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INTRODUCTION

This report presents the results of our investigation of subsurface conditions (1) along the alignment of a proposed drainage tunnel and (2) adjacent to each of four existing box culverts which are scheduled for reconstruction. Our investigation is part of an overall redesign of a drainage channel running through Pearl City, Oahu, Hawaii. The locations of this drainage channel, the proposed tunnel alignment and the four box culverts are shown on the Map of Project, Plate 1.

Our investigation may be separated into two parts - the tunnel alignment and the box culverts, and this report is divided into two sections accordingly. The first section pertains to the investigation along the proposed tunnel alignment, and the second pertains to the investigation at the box culvert sites.

Details of our field exploration and laboratory testing procedures are provided in the appendix attached to this report.

PROPOSED DRAINAGE TUNNEL

SUMMARY

Approximately the first half of the proposed tunnel alignment will be at relatively shallow depths and can be constructed by cut and cover methods. The second half will require through-tunnelling.

The cut and cover section will be located primarily in clayey silt soils, but hard rock will be encountered in the second half of this section of alignment. Walls for this section of the tunnel may be designed for an equivalent fluid pressure of 50 pounds per square foot per foot of depth below ground surface. The soil possesses expansive characteristics, and precautions should be taken during design and construction to provide for possible expansion pressures on the bottom of the first half of this section of tunnel.

The through-tunnel section will be in basaltic rock. The rock is broken by bedding planes and joints and is weathered. Some local stability problems may be expected during tunnelling. Roof support should be designed to resist maximum pressures of 1,500 psf. Normal pressures are expected to range from 500 to 1,500 psf, depending on the type of lining used. Lining could consist of reinforced gunite or corrugated metal liner plate. Roof support could also be provided by rock bolts.

PROJECT CONSIDERATIONS

The proposed drainage tunnel will provide additional capacity for the drainage channel shown on Plate 1. The proposed alignment will connect the existing channel at Kamehameha Highway in front of the Pearl City Shopping Center to a point on the channel downstream from its intersection with Hoomalu Street. The approximate location of the proposed alignment is shown on the Proposed Tunnel Plan and Profile, Plate 2.

The lower section of the tunnel will be essentially an extended box culvert and can be constructed by cut-and-cover methods. This section of the tunnel will extend from Station 0+00 to approximately Station 6+00. The interior tunnel dimensions will be on the order of 10 feet high and 12 to 18 feet wide. The primary purpose of our investigation along this section of tunnel alignment was to determine lateral earth pressures for use in design of the tunnel walls and to evaluate excavation conditions.

The through-tunnel section will extend from approximately Station 6+00 to Station 12+00. The inside dimension of this section of the tunnel will be approximately 11 feet in diameter. The floor and walls of the tunnel will require lining for optimum flow conditions. Overhead support of the tunnel will be of great importance due to the existence of a residential area above the tunnel. For this same reason, stability of the portal excavations during construction will also be of importance.

SITE DESCRIPTION

The proposed tunnel alignment passes through the parking lot in front of the Pearl City Shopping Center from Station 0+00 to Station 4+00. The alignment then runs along Puu Momi

Street to the intersection with Puu Kula Drive at approximately Station 8+00. From Station 8+00 to the upstream portal at Station 12+00, the alignment passes under a residential area.

Investigation of subsurface conditions was performed along the proposed tunnel alignment by drilling five borings at locations shown on Plate 2. Soils in the tunnel area consisted of 15 to 20 feet of red-brown clayey silt in the form of residual soil and decomposed basalt. This soil layer may be locally thicker, since nearly 30 feet of it was encountered at Boring 5. Part of the parking lot area rests on clayey silt fill near Boring 1, where six or seven feet of this material was observed.

The soils rest on bedrock consisting of weathered vesicular basalt. This rock was deposited by lava flows on the southwest flank of the now-extinct Koolau Volcano. The lava flowed on a slope of approximately eight to ten degrees, and bedding planes encountered by the tunnel are expected to be at about this inclination. The direction of flow was from the northeast, nearly parallel to the tunnel alignment. Quality of the rock ranges from highly jointed and weathered to fresh and massive. Most joints are nearly horizontal, but many are at steep inclinations, breaking the rock mass into blocks. Often the joints are coated by a thin layer of clay or silt. Closely jointed zones can be as much as five or six feet thick. Additionally, buried soil layers of at least two feet in thickness were encountered by our borings. Logs of the borings in the tunnel area are presented in Plates A-2A through A-2E in the Appendix and may be referred to for more detail on the subsurface conditions at the boring locations.

No water was encountered in any of the borings during drilling, and the permanent ground water table is believed to lie near sea level, well below the invert level of the tunnel.

CONCLUSIONS AND RECOMMENDATIONS

Cut and Cover Section - This section of the tunnel will be constructed primarily in clayey silt soils, including some artificial fill. Beyond about Station 3+00, the tunnel invert will be located in basaltic bedrock. The entire tunnel height may be in rock at some point between Stations 5+00 and 6+00.

The walls of the cut and cover section will be relatively rigid so that lateral earth pressures may become fairly large. We recommend that the walls be designed for an equivalent fluid pressure of 50 pounds per square foot per foot of depth below the ground surface.

An expansion test performed on a sample of the soil obtained from the vicinity of the tunnel invert indicated pronounced expansive characteristics. Therefore, we expect that the soils could exert vertical expansion pressures of at least

1,000 pounds per square foot if they were allowed access to water, and we recommend that the floor of the tunnel be designed for these conditions. Additionally, we recommend that during construction, the floor of the tunnel be poured upon a granular cushion and that 24 hours prior to the pour, this cushion and the underlying soil be thoroughly saturated. The purpose of the saturation will be to allow the underlying clayey silt to expand initially and thus relieve a portion of the potential expansion.

Granular backfill should be placed between the excavation walls and the tunnel. We recommend that this backfill be compacted to about 90 percent of its maximum dry density as determined by AASHO Test Method T-180.

Excavation of the soils for this section of the tunnel may be by conventional means. The walls of the excavation may require bracing to avoid caving, particularly if the cuts are vertical and are allowed to stand open for a prolonged period of time. Beyond Station 3+00 we expect that excavation will be partially in rock. Blasting would be required for fastest construction progress but may be objectionable because of possible damage to surrounding structures. Pneumatic equipment may be preferable, although excavation would proceed at a slower rate. The relocation of some buried utilities adjacent to this section of the tunnel probably will be required.

<u>Through-Tunnel Section</u> - Through-tunnelling should commence and end at points where there is at least five feet of rock cover above the anticipated roof of the tunnel excavation. We expect that these points will be near Stations 6+00 and 12+00.

Since the gradient of the tunnel is flatter than the attitude of the basalt flows, some instability in the rock will result at intersections of the rock bedding planes with the tunnel roof. Local caving could be prevented by installation of roof support as quickly after excavation as possible.

Because of the jointed and weathered nature of the rock, we believe that average roof pressures on the tunnel lining would be equivalent to about five feet of rock or 850 pounds per square foot. However, at locations where weaker fractured zones and buried soil layers are above the tunnel roof, pressures could locally reach at least 1,500 pounds per square foot. The rock is so variable in quality that these weaker zones cannot be predicted with any degree of confidence. Therefore, we recommend that all roof support be designed for pressures of 1,500 psf.

Support of the tunnel roof probably will be provided by a tunnel lining extending around the full perimeter of the tunnel. Two types of lining are suggested in a subsequent paragraph. As an alternative, roof support could be provided by rock bolts. These bolts could be placed immediately after each mucking round of the excavation cycle. They could provide permanent roof support if grouted after the completion of the excavation. If rock bolts were used, we would recommend that they be placed on a square grid pattern which would provide three or four bolts across the width of the roof. The rock bolts would have to be on the order of eight feet long and be manufactured of moderate to high-strength steel. One-foot square bearing plates would be used in conjunction with the rock bolts. In the worst zones of fractured rock, wire mesh could also be used to restrain minor rockfall. If rock bolts were specified, installation should be under the supervision of a qualified engineer.

We believe that normal pressures of between 500 and 1,500 pounds per square foot may ultimately be applied along the sides of the tunnel lining. The magnitude may depend in part on the type of lining used, with the larger pressures applied to flexible lining such as corrugated steel tunnel plate. Lateral pressures on a rigid lining such as reinforced gunite probably would be of smaller magnitude. Regardless of the type of lining used, all voids between the lining and the tunnel walls should be filled by backpacking with sand, grouting or a combination of the two.

Excavation of the through-tunnel section presumably will not be by blasting because of the residential neighborhood overhead. Pneumatic excavation probably will require a longer construction period but may be more desirable from other considerations. If blasting were used, however, we recommend that smooth blasting techniques be considered as a construction requirement. Such techniques as pre-splitting would minimize overbreak and damage to the rock in the tunnel walls and roof. This would contribute to lower roof and wall pressures than if normal blasting procedures were used, and should add some safety margin to the design of the tunnel lining and roof support.

We recommend that the tunnel be constructed from its southwest portal so that feather-edges between bedding planes and the tunnel roof will be encountered head-on at the excavation face rather than from behind. Any caving at these intersections then would tend to take place in smaller segments at the face rather than in large blocks after the face had reached an intersection. Construction from the southwest portal would also provide a slight downward gradient for removing excavated material and would allow the existing drainage channel to continue in normal use until the tunnel is nearly complete.

We believe that only minor seepage will be encountered during tunnelling because the permanent water table apparently exists at an elevation much lower than the proposed tunnel invert.

At the southwest portal of the tunnel near Station 6+00, stability of the cuts in soil should be assisted by bracing or by laying the slopes back at least to 1:1, whichever is more feasible. The approach used probably would depend on the exact location of the portal relative to the adjacent residential area. Presumably all soils removed from above the tunnel will be replaced at the end of construction. Provided that the soil is placed back in a well-compacted condition, long-term slope stability at the portal then should not be a problem. The northeast portal should require no excavation into soil slopes, and slope stability should remain essentially unchanged from present conditions.

PROPOSED NEW BOX CULVERTS

SUMMARY

Investigation of subsurface conditions at the sites of four proposed new box culverts was conducted to determine lateral earth pressures for use in design of the new culvert walls. Design lateral earth pressures should be an equivalent fluid pressure of 50 pounds per square foot per foot of depth below the ground surface. Soils encountered near the box culvert inverts appear to be non-expansive.

PROJECT CONSIDERATIONS

Double box culverts now exist along the lined drainage channel at points where it intersects streets in the Pearl City area. The locations of the drainage channel and the culverts investigated are shown on the Map of Project, Plate 1. This portion of the project will involve replacement of the existing box culverts by new single-opening culverts. Inside dimensions of the new culverts apparently will vary but will be on the order of 10 to 11 feet high and 17 to 20 feet wide. The primary purpose of our investigation was to determine lateral earth pressures which should be used for design of the culvert walls.

SITE DESCRIPTION

The drainage channel in question probably occupies a natural drainage course. In-place soils along this drainage are now obscured by the channel lining and by adjacent residential development.

Each box culvert was investigated by drilling two borings, one on each side near opposite ends of the existing culvert. Materials encountered consisted primarily of old backfill composed of reddish clayey silt, occasionally mixed with gravel or boulders. This fill usually changed at depth to in-place decomposed basalt in the form of clayey silt. Although the areas of investigation presumably were within an old natural drainage channel, no signs of alluvial soils were noted, with the possible exception of some of the boulders encountered. Details of subsurface conditions at each boring are presented graphically in the Log of Borings, Plates A-2F through A-2M. No water or hard rock was encountered in any of the borings during drilling.

CONCLUSIONS AND RECOMMENDATIONS

Subsurface material encountered adjacent to the existing culvert was primarily fill, presumably placed during construction of the culverts.

Because of the relative rigidity of the culvert walls, we recommend that an equivalent fluid pressure of 50 pounds per square foot per foot of depth below ground surface may be used for lateral earth pressures in design of the walls.

Expansion tests conducted on two samples of decomposed basalt from near invert elevations of the culverts indicated that the material was not expansive, even after air-drying. Therefore, design of the culvert floors against expansion does not appear appropriate. However as a precaution, we recommend that the floor of each culvert be poured on a granular cushion and that the cushion and underlying soil be saturated 24 hours prior to the pour. Granular backfill should be placed between the excavation walls and the new culverts. We recommend that this backfill be compacted to about 90 percent of its maximum dry density as determined by AASHO Test Method T-180.

Excavation along the sides of the culvert can be made by conventional means since most of the material removed probably will be fill. The existence of some boulders within the fill should be noted, and these could provide minor excavation difficulties. The excavation slopes should be laid back to at least 1:1 for temporary stability during construction. In some cases, poorly compacted old backfill may have to be removed completely.

Numerous underground utility lines parallel to and crossing the drainage channels at the culverts will complicate excavation and construction of the new culverts.

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The following Plates and Appendix are attached and complete this report:

Plate 1	-	Map of Project
Plate 2	-	Proposed Tunnel Plan and Profile
Appendix	-	Field Exploration and Laboratory Testing

Respectfully Submitted,

DAMES & MOORE

David C. Liu

DCL DRR jms

DAVID C. LIU
REGISTERED PROFESSIONAL ENGINEER
No. 1265
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

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BY W Buss DATE 6-11-70
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REVISIONS
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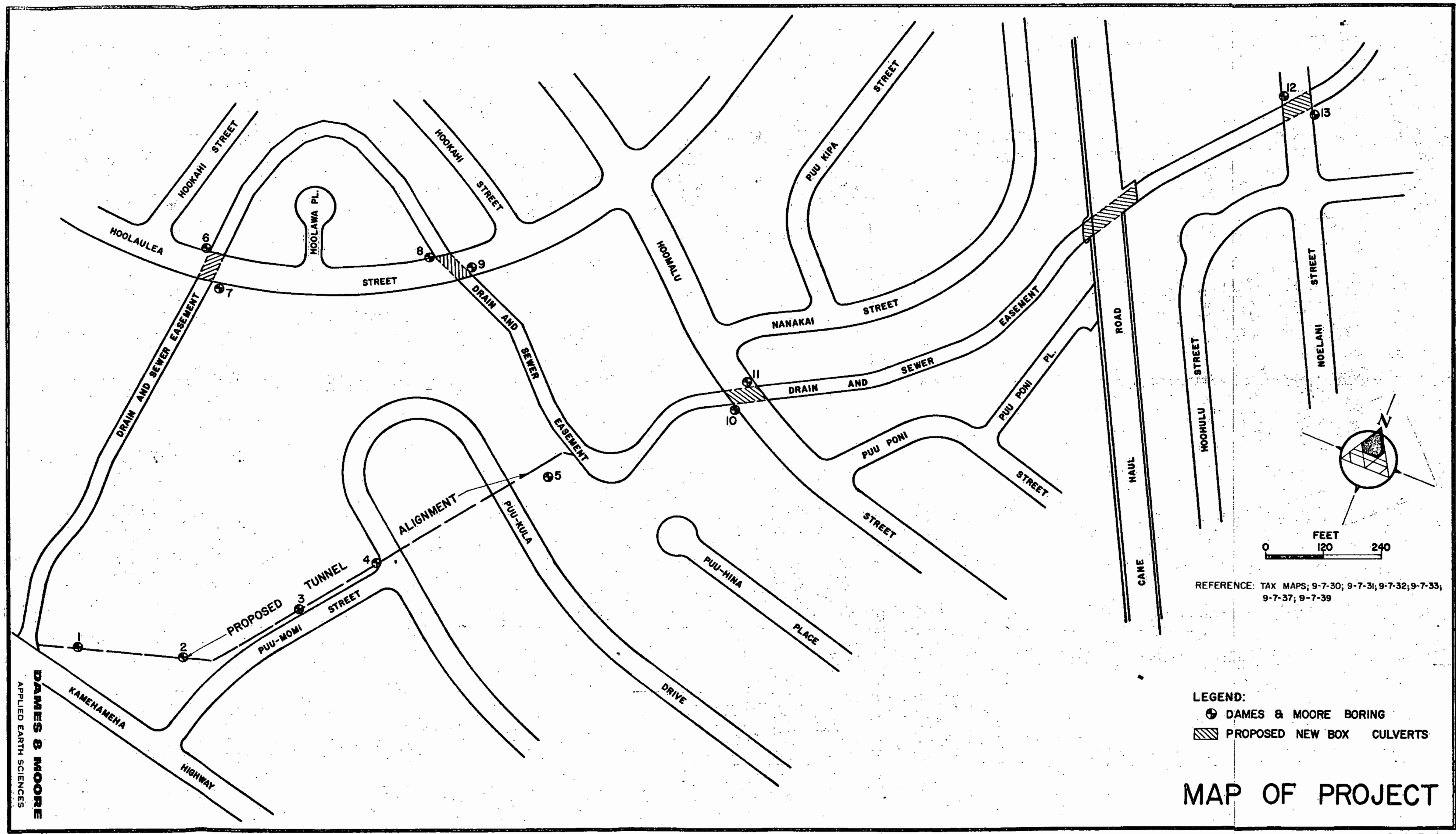

MAP OF PROJECT

PLATE I

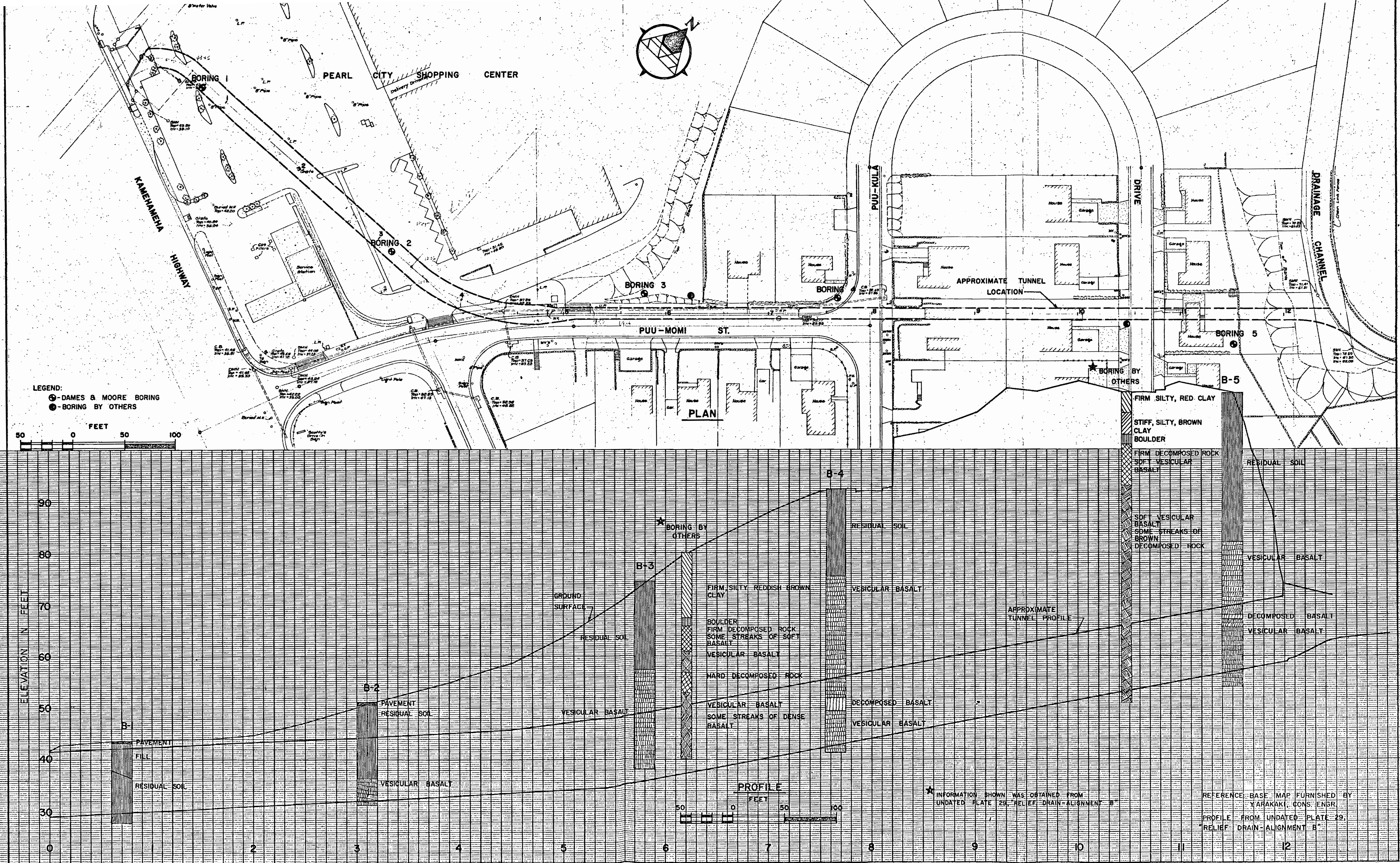

PLATE 2
PROPOSED TUNNEL PLAN AND PROFILE

APPENDIX

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

Our field exploration commenced on August 3 and was completed on August 14, 1970. Investigation was conducted by means of 13 borings, 5 along the proposed tunnel alignment and 8 at the proposed new box culverts. Two borings were made at each box culvert, one near each end and on opposite sides of the culvert. Borings along the proposed tunnel alignment ranged in depth from 16.0 to 57.5 feet. Boring depths at the culvert locations ranged from 16.0 to 20.5 feet. The borings were drilled by subcontracted personnel using a truck-mounted rotary drill rig. The borings were advanced by a five-inch continuous flight auger in soil and a four-inch diamond core bit in rock. A smaller skid-mounted drill rig was required at Boring 5, because of the location in the backyard of a residence near the upstream portal of the proposed tunnel. All rights of entry onto private property were obtained by the City and County of Honolulu.

Drilling was performed under the general supervision of one of our engineers. Soil samples were obtained at about four- or five-foot intervals using equipment described in Exhibit A-1. The samples were returned to our laboratory for subsequent examination and testing. Rock cores were obtained as a matter of course during the rock drilling, and all core was also returned to our laboratory for examination and testing. The core is available for viewing if desired. Four-inch diameter core was required in order to achieve as complete recovery of core as possible.

All soil and rock samples were examined by our engineer in the field, and the soils were classified in accordance with the Unified Soil Classification System described on Plate A-1. Our engineer also maintained logs during our drilling. These are presented in Plates A-2A through A-2M.

The steep slope at the upstream portal area of the proposed tunnel was examined by one of our geological engineers, and the rock exposed was roughly correlated with that encountered in Boring 5, that nearest the upstream portal. It appeared that rock along the proposed alignment extended to higher elevations than at Boring 5.

LABORATORY TESTING

GENERAL

A variety of tests were performed on soil and rock samples to determine the quality of the material and specific engineering properties. Tests conducted included moisture-density determinations, unconfined and triaxial compression tests, direct shear strength tests, expansion tests and Atterberg limits determinations.

SOIL TESTS

Moisture-Density Tests - These tests were conducted on most of the relatively undisturbed soil samples, including those used for strength testing. Moisture content and dry density for all samples so tested are shown on the Log of Borings, Plates A-2A through A-2M.

Triaxial Strength Tests - These tests included both unconfined and confined tests. Triaxial tests were conducted under unconsolidated-undrained conditions. The test method and equipment is described on Exhibit A-2. Results of the testing follow.

Boring No.	Depth (Ft.)	Confining Pressure (psf)	Maximum Deviator Stress (psf)
1	13.0	1000	19,700+*
1	15.0	0	8350
3	8.5	1000	3750
4	11.0	1500	4750
4	15.5	2000	6200
5	20.75	0	2600
5	26.25	0	3250
6	12.5	1500	2425
10	3.0	500	2725
11	8.0	1000	2425
11	12.0	1000	2100
11	20.0	1000	2100
12	4.5	500	2350
12	18.5	3000	4150
13	3.5	0	8050
13	17.5	2000	4350

*Strength of sample exceeded capacity of testing machine.

Direct Shear Tests - These tests were conducted to complement triaxial tests when available samples were not of sufficient length for triaxial testing. The method of testing and equipment used are explained on Exhibit A-3. Results of these tests follow.

Boring No.	Depth (Ft.)	Normal Pressure (psf)	Maximum Shear Stress (psf)
1	2.0	2000	4500
2	4.5	500	4200
2	8.5	1000	2200
3	2.5	500	1560
3	5.0	1500	3300
4	3.5	1000	2850

Expansion Tests - Three of these tests were conducted to determine the tendency of in-place soils to swell when exposed to excess moisture. The materials so tested were those in the vicinity of the inverts of the cut and cover section of the tunnel and the box culverts. The samples were air-dried for one day to duplicate conditions during construction. They were then saturated under surcharge loads of 400 or 500 pounds per square foot. The amount of vertical expansion was observed and recorded. Results of the tests follow.

Boring No.	Depth (Ft.)	Surcharge Load (psf)	Expansion (% of air-dried height)
2	12	500	6.0
7	15	400	-0.8
12	15	400	-1.1

Atterberg Limits - The Atterberg limits were determined for one sample to determine its soils classification as a guide in classifying other samples. Test results were:

Boring No.	Depth (Ft.)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Classification
1	12.5	51	33	18	MH-ML

ROCK TESTS

Density Determinations - Eight sections of rock core were selected for density measurements. The core selected provided a range of degree of vesicularity and severity of weathering. The results of these tests are shown on the Log of Borings, A-2C through A-2E.

Unconfined Compression Tests - These tests were conducted by a subcontracted testing laboratory on six of the core samples selected for density determinations. The results of the tests are considered as maximum rock strengths and are the upper limits which could be used in design of tunnel lining and supports. The rock generally is sufficiently jointed so that any movement would probably occur along the joints rather than through intact rock similar to that tested. Results of the tests follow.

Boring No.	Depth (Ft.)	Maximum Compressive Stress (psi)
3	20.0	9670
4	34.0	1530
4	35.5	4540
4	46.0	3940
5	31.0	1440
5	42.0	9960

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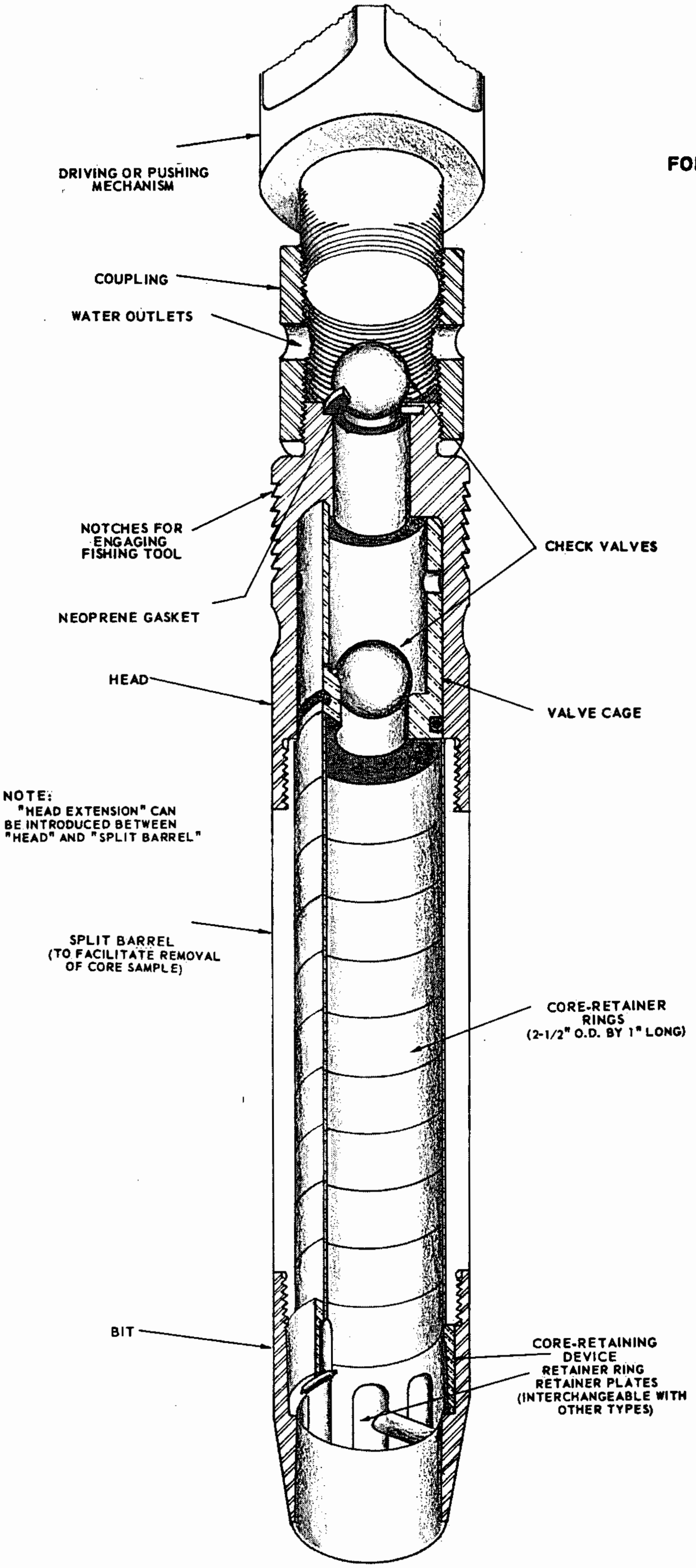
The following Exhibits and Plates are attached and complete this Appendix:

Exhibit A-1	-	Soil Sampler Type U
Exhibit A-2	-	Method of Performing Unconfined Compression and Triaxial Compression Tests
Exhibit A-3	-	Method of Performing Direct Shear and Friction Tests
Plate A-1A	-	Unified Soil Classification System
Plate A-2A	-	Log of Borings, Boring 1
Plate A-2B	-	Log of Borings, Boring 2
Plate A-2C	-	Log of Borings, Boring 3
Plate A-2D	-	Log of Borings, Boring 4
Plate A-2E	-	Log of Borings, Boring 5
Plate A-2F	-	Log of Borings, Boring 6
Plate A-2G	-	Log of Borings, Boring 7
Plate A-2H	-	Log of Borings, Boring 8
Plate A-2I	-	Log of Borings, Boring 9
Plate A-2J	-	Log of Borings, Boring 10
Plate A-2K	-	Log of Borings, Boring 11
Plate A-2L	-	Log of Borings, Boring 12
Plate A-2M	-	Log of Borings, Boring 13

EXHIBIT A-1

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER



ALTERNATE ATTACHMENTS

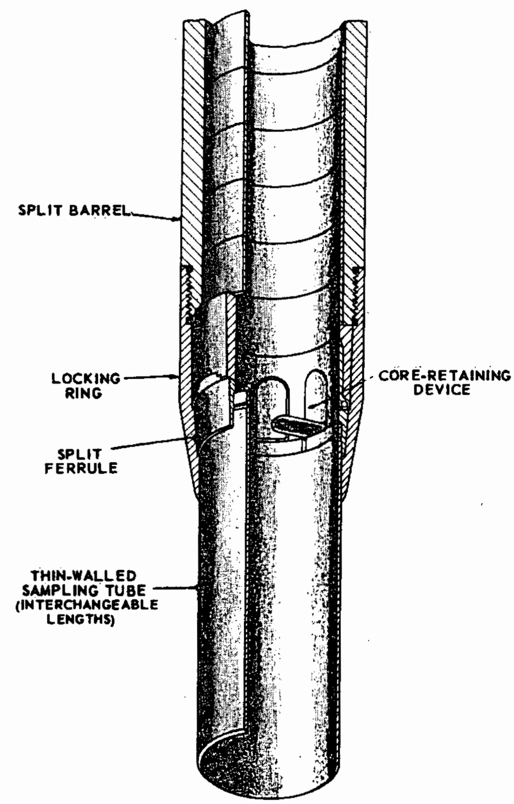


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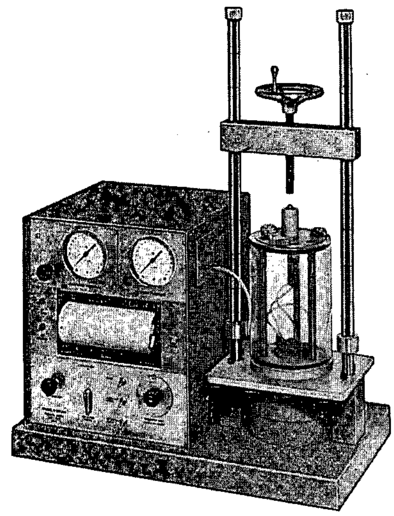
EXHIBIT A-2

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.



TRIAXIAL COMPRESSION TEST UNIT

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

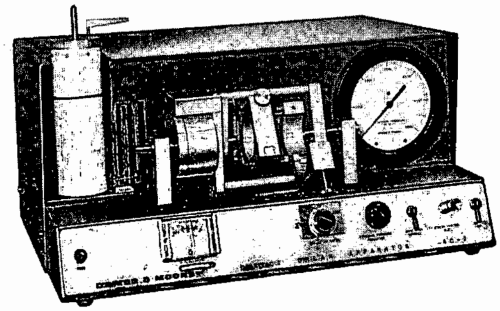
DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

EXHIBIT A-3

Method Of Performing Direct Shear and Friction Tests

Direct shear tests are performed to determine the shearing strengths of soils. Friction tests are performed to determine the frictional resistances between soils and various other materials such as wood, steel, or concrete. The tests are performed in the laboratory to simulate anticipated field conditions.



DIRECT SHEAR TESTING & RECORDING APPARATUS

Each sample is tested within three brass rings, two and one-half inches in diameter and one inch in length. Undisturbed samples of in-place soils are tested in rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.

Direct Shear Tests

A three-inch length of the sample is tested in direct double shear. A constant pressure, appropriate to the conditions of the problem for which the test is being performed, is applied normal to the ends of the sample through porous stones. A shearing failure of the sample is caused by moving the center ring in a direction perpendicular to the axis of the sample. Transverse movement of the outer rings is prevented.

The shearing failure may be accomplished by applying to the center ring either a constant rate of load, a constant rate of deflection, or increments of load or deflection. In each case, the shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soil is determined from the resulting load-deflection curves.

Friction Tests

In order to determine the frictional resistance between soil and the surfaces of various materials, the center ring of soil in the direct shear test is replaced by a disk of the material to be tested. The test is then performed in the same manner as the direct shear test by forcing the disk of material from the soil surfaces.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			*GRAPH SYMBOL*	*LETTER SYMBOL*	*TYPICAL DESCRIPTIONS*
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50 % OF COARSE FRACTION <u>RETAINED</u> ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50 % OF MATERIAL IS <u>LARGER</u> THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION <u>PASSING</u> NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT <u>LESS</u> THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT <u>GREATER</u> THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
MORE THAN 50 % OF MATERIAL IS <u>SMALLER</u> THAN NO. 200 SIEVE SIZE				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICTY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTES:

1. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE CLASSIFICATIONS.
2. WHEN SHOWN ON THE BORING LOGS, THE FOLLOWING TERMS ARE USED TO DESCRIBE THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE COMPACTNESS OF COHESIONLESS SOILS.

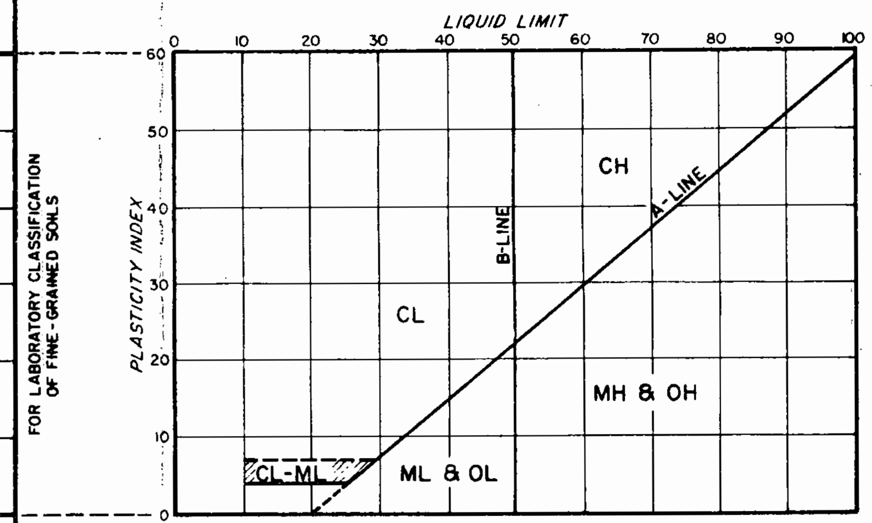
COHESIVE SOILS	(APPROXIMATE SHEARING STRENGTH IN KSF)	COHESIONLESS SOILS	
VERY SOFT	LESS THAN .25	VERY LOOSE	THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE, AND SOIL DENSITY DATA.
SOFT	0.25 TO 0.5	LOOSE	
MEDIUM STIFF	0.5 TO 1.0	MEDIUM DENSE	
STIFF	1.0 TO 2.0	DENSE	
VERY STIFF	2.0 TO 4.0	VERY DENSE	
HARD	GREATER THAN 4.0		

GRADATION CHART

MATERIAL SIZE		*PARTICLE SIZE* LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE✱	MILLIMETERS	SIEVE SIZE✱
SAND	FINE	.074	#200✱	0.42	#40✱
	MEDIUM	0.42	#40✱	2.00	#10✱
	COARSE	2.00	#10✱	4.76	#4✱
GRAVEL	FINE	4.76	#4✱	19.1	3/4"•
	COARSE	19.1	3/4"•	76.2	3"•
COBBLES		76.2	3"•	304.8	12•
BOULDERS		304.8	12•	914.4	36"

✱ U.S. STANDARD • CLEAR SQUARE OPENINGS

PLASTICITY CHART



SAMPLES

- INDICATES UNDISTURBED SAMPLE
- INDICATES DISTURBED SAMPLE
- INDICATES SAMPLING ATTEMPT WITH NO RECOVERY
- INDICATES LENGTH OF CORING RUN

NOTE:
DEFINITIONS OF ANY ADDITIONAL DATA REGARDING SAMPLES ARE ENTERED ON THE FIRST LOG ON WHICH THE DATA APPEAR.

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

BORING 1

SURFACE ELEVATION 44 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	GRAPH SYMBOL	LETTER SYMBOL	DESCRIPTION
								2" ASPHALTIC CONCRETE
							GM	6" CRUSHED BASALT BASE COURSE
							MH	RED-BROWN CLAYEY SILT (FIRM TO STIFF, FILL)
27.4	78.4	12		▩				
					5			
							MH ML	RED-BROWN CLAYEY SILT (STIFF, RESIDUAL SOIL)
28.1	98.2	14		▩				
					10			
								BECOMES BROWN AND HARD WITH SOME GRANULAR TEXTURE (POSSIBLY DECOMPOSED ALLUVIUM)
26.8	92.6	44		▩				
28.9	80.7	45		▩	15			
								BORING COMPLETED AT 16.0 FEET ON 8-6-70
								NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
□ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2A

BORING 2

SURFACE ELEVATION 51 ± FEET
MSL DATUM

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Letter Symbol	Description
						GM	2", ASPHALTIC CONCRETE 6" CRUSHED BASALT BASE COURSE
						MH	RED-BROWN CLAYEY SILT (VERY STIFF, RESIDUAL SOIL)
24.6	102.5	28		■	5		
28.0	82.7	14		■			
					10		
26.9	94.5	44/6"		■			BECOMES BROWN AND HARD
			NX 100%	I	15		VESICULAR OLIVINE BASALT, SLIGHTLY WEATHERED, UNJOINTED.
					20		BECOMING MORE VESICULAR AT 20 FEET BORING COMPLETED AT 20.0 FEET ON 8-7-70 NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

■ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
□ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2B

REVISIONS BY ______ DATE ______
FILE 4402-037
DATE 70
CHECKED BY
66.7 (REV. 6-61)

466.12 (4-64)

FILE 4402-037
BY W. Bun DATE 9-1-70
CHECKED BY ______ DATE ______

REVISIONS
BY ______ DATE ______
BY ______ DATE ______
PLATE ______ OF ______

BORING 3

SURFACE ELEVATION 74 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
					0	MH	DARK RED CLAYEY SILT (VERY STIFF TO HARD, RESIDUAL SOIL)
25.0	76.0	27		▩			
25.3	81.7	42		▩	5		
33.8	69.3	20		▩		MH	GRADES TO GRAY AND RED-BROWN CLAYEY SILT (VERY STIFF TO HARD, DECOMPOSED BASALT)
					10		
34.5	75.0	57/9"		▩			
					15		
		60		☒			GRAY VESICULAR BASALT, WEATHERED IN TOP 12 INCHES. JOINTED HORIZONTALLY AT 3 TO 12 INCH INTERVALS. NO VERTICAL JOINTS IN CORE. SOME OXIDATION ON JOINTS.
	147.6		4" 85%	I	20		DRILLING WATER CIRCULATION LOST AT 21 FEET
			4" 100%	I			CLOSELY JOINTED VESICULAR BASALT. MODERATELY WEATHERED.
							WEATHERED VESICULAR BASALT. JOINTED HORIZONTALLY AT 3 TO 7 INCH INTERVALS. ALSO JOINTED VERTICALLY.
					25		CLOSELY JOINTED VESICULAR BASALT. HIGHLY WEATHERED.
			4" 100%	I			WEATHERED VESICULAR BASALT. JOINTED HORIZONTALLY AT 3 TO 10 INCH INTERVALS. ALSO MANY VERTICAL JOINTS. VERY WEATHERED ON MOST JOINTS.
			4" 100%	I	30		
	114.1			I	35		SLIGHTLY WEATHERED VESICULAR BASALT. JOINTED HORIZONTALLY AT 5 TO 12 INCH INTERVALS. SOME VERTICAL JOINTS. WEATHERED ON HORIZONTAL JOINTS.

BORING COMPLETED AT 36.5 FEET ON 8-4-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:
▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY- 300 -LB WEIGHT DROPPING 36 INCHES

DAMES & MOORE

PLATE A-2C

466.12 (4-64)

FILE 4402-037
BY W Bun DATE 9-1-70
CHECKED BY ______ DATE ______

REVISIONS
BY ______ DATE ______
BY ______ DATE ______
PLATE ______ OF ______

BORING 4

SURFACE ELEVATION 92 ± FEET
MSL DATUM

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Graph Symbol	Letter Symbol	Description
24.7	74.2	15		▩			MH	SMALL ROOTS NEAR SURFACE DARK RED CLAYEY SILT (STIFF TO HARD, RESIDUAL SOIL)
25.3	81.8	57		▩	5			
31.6	88.2	30		▩			MH	GRADES TO BROWN AND GRAY CLAYEY SILT, OCCASIONALLY WITH GRANULAR TEXTURE (STIFF TO HARD, DECOMPOSED BASALT)
35.9	75.5	20		▩	10			
32.4	78.0	15/9"		▩	15			
			4" 100%	I	20			GRAY VESICULAR BASALT, HIGHLY WEATHERED AT TOP. JOINTED HORIZONTALLY AT 3 TO 10 INCH INTERVALS WITH HIGHLY JOINTED ZONES AT 20 AND 22 FEET. WEATHERED ON ALL JOINTS.
			4" 20%	I	25			VERY POOR CORE RECOVERY INDICATES HIGHLY WEATHERED AND JOINTED BASALT
			4" 60%	I				
			4" 100%	I	30			WEATHERED VESICULAR BASALT, JOINTED HORIZONTALLY AT 3 TO 6 INCHES INTERVALS. ALSO JOINTED VERTICALLY. VERY WEATHERED ON JOINTS.
								GRAY VESICULAR BASALT, MORE MASSIVE, JOINTED HORIZONTALLY AT 8 TO 20 INCH INTERVALS. WEATHERED ON JOINTS.
	117.2		4" 100%	I	35			
			4" 100%	I				DRILLING WATER CIRCULATION LOST AT 36.5 FEET HIGHLY WEATHERED AND JOINTED VESICULAR BASALT. LESS JOINTED AT 38 AND 40 FEET
					40			
			4" 100%	I			ML	RED-BROWN SANDY SILT (STIFF, DECOMPOSED BASALT)
					45			GRADES TO WEATHERED VESICULAR BASALT, JOINTED HORIZONTALLY AT 4 TO 10 INCH INTERVALS, SOME VERTICAL JOINTING. HIGHLY JOINTED ZONES AT 44.5 AND 46.5 FEET
	109.5		4" 95%	I				GRAY VESICULAR BASALT, JOINTED HORIZONTALLY AT 4 TO 16 INCH INTERVALS. LITTLE WEATHERING.
					50			BORING COMPLETED AT 51.0 FEET ON 8-6-70 NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY- 300 -LB WEIGHT DROPPING 36 INCHES

DAMES & MOORE

PLATE A-2D

FILE 4402-037
BY W.Bun DATE 9-1-70
CHECKED BY ___ DATE ___

REVISIONS
BY ___ DATE ___
BY ___ DATE ___
PLATE ___ OF ___

BORING 5

SURFACE ELEVATION 111 ± FEET
MSL DATUM

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Depth in Feet	Letter Symbol	Description
					MH	RED-BROWN CLAYEY SILT (LOOSE TO FIRM, FILL)
33.7	77.6	50/6"		5	MH	BROWN CLAYEY SILT (VERY STIFF TO HARD, DECOMPOSED BASALT)
33.3	72.1	120/10"		10		COLOR CHANGES TO RED AND GRAY
				15		
35.4	83.1	100/7"				
39.9	75.0	120/7"		20	MH-ML	GRAY CLAYEY SILT, SOMETIMES WITH GRANULAR TEXTURE (HARD, DECOMPOSED TO HIGHLY WEATHERED BASALT)
42.7	69.4	95/8"		25		
						WEATHERED VESICULAR BASALT
	82.4	100/2"	4" 100%	30		GRAY VESICULAR BASALT JOINTED HORIZONTALLY AT 4 TO 11 INCH INTERVALS. ALSO JOINTED VERTICALLY. WEATHERED ON JOINTS
			4" 100%	35		
			4" 95%			WEATHERED VESICULAR BASALT JOINTED HORIZONTALLY AT 3 TO 8 INCH INTERVALS AND JOINTED VERTICALLY. HIGHLY WEATHERED ON MOST JOINTS
	155.2			40		GRAY DENSE TO VESICULAR BASALT, MASSIVE, JOINTED HORIZONTALLY AT 6 TO 20 INCH INTERVALS. SOME WEATHERING ON JOINTS.
	94.7		4" 95%		ML	RED-BROWN SILT (DECOMPOSED BASALT)
						HIGHLY WEATHERED VESICULAR BASALT, CLOSELY JOINTED
				45		SLIGHTLY WEATHERED VESICULAR BASALT JOINTED HORIZONTALLY AT 6 TO 12 INCHES INTERVALS
			4" 95%			HIGHLY WEATHERED AND JOINTED VESICULAR BASALT
				50		GRAY VESICULAR BASALT JOINTED HORIZONTALLY AT 5 TO 43 INCH INTERVALS. ALSO SOME DIAGONAL JOINTS. SLIGHTLY WEATHERED ON JOINTS
			4" 100%	55		
						BORING COMPLETED AT 57.5 FEET ON 8-14-70 NO WATER ENCOUNTERED

NOTES:
- ▨ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
- ☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
- ☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
- I -DEPTH AND LENGTH OF CORE RUN

DRIVING ENERGY- 140 -LB WEIGHT DROPPING 36 INCHES

LOG OF BORINGS

PLATE A-2E

BORING 6

SURFACE ELEVATION 55 ± FEET
MSL DATUM

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Graph Symbol	Letter Symbol	Description
							MH	Red-brown clayey silt (loose and dry in top several feet, fill)
		10		⊠				
					5			
		6		■				(becomes moist and soft to stiff, fill)
					10			
30.0	95.4	10		■				
					15			
39.5	76.3	4		■				
							GM-MH	Mixture of red-brown clayey silt and crushed rock fragments (firm to stiff, fill)
22.8	93.3	35		■				

Boring completed at 19.0 feet on 8-10-70
No water encountered

LOG OF BORINGS

NOTES:

- ■ -depth at which undisturbed sample was taken
- ⊠ -depth at which disturbed sample was taken
- ☐ -depth at which sample was lost during extraction
- I -depth and length of core run

Driving energy - 300-lb weight dropping 30 inches

BORING 7

SURFACE ELEVATION 56 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
							3", ASPHALTIC CONCRETE
						MH	DARK RED CLAYEY SILT WITH A FEW ROCK FRAGMENTS (FIRM TO STIFF, FILL)
31.2	88.7	12		■	5		
30.3		12		■	10	MH	RED-BROWN CLAYEY SILT (STIFF, RESIDUAL SOIL)
							BOULDER AT 14'
41.1	70.2	26		■	15	MH-ML	RED AND GRAY CLAYEY SILT (HARD, DECOMPOSED BASALT)
19.5	94.0	59		■			GRADING TO HIGHLY WEATHERED VESICULAR BASALT

BORING COMPLETED AT 19.0 FEET ON 8-7-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

- ■ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
- ⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
- □ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
- I -DEPTH AND LENGTH OF CORE RUN

DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2G

REVISIONS BY ____ DATE ____
FILE 4402-037
BY ____ CHECKED BY DRR DATE ____
946.7 (REV. 6-61)

BORING 8

SURFACE ELEVATION 68 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
						MH	RED-BROWN CLAYEY SILT (HARD, PROBABLY FILL)
23.3	79.9	26		■			
					5		
							BOULDER AT 7'
25.5	81.1	16		■			
					10		BOULDERS. UNABLE TO SAMPLE (PROBABLY FILL)
					15		
7.2	115.8	44/8"		■		SM-ML	GRAY SILTY SAND AND SANDY SILT (HARD, PROBABLY FILL)

BORING COMPLETED AT 17.0 FEET ON 8-10-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

- ■ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
- ⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
- □ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
- I -DEPTH AND LENGTH OF CORE RUN

DRIVING ENERGY -300 -LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2H

REVISIONS BY ____ DATE ____
FILE 4402-037
BY ____ DATE ____ CHECKED BY ____
946.7 (REV. 6-61)

BORING 9

SURFACE ELEVATION 68 ± FEET
DATUM MSL

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	LETTER SYMBOL	DESCRIPTION
							6", ASPHALTIC CONCRETE AND BASE COURSE
						MH-GM	RED AND BROWN CLAYEY AND SANDY SILT WITH MANY ROCK FRAGMENTS (HARD, FILL)
21.8		7/6"		⊠			
25.7	89.0	14		■	5		
18.9		20		■	10		BOULDER AT 11'
							GRAY AND ORANGE HIGHLY WEATHERED BASALT (HARD, POSSIBLY A BOULDER)
41.0	70.0	50		■	15		

BORING COMPLETED AT 16.0 FEET ON 8-10-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

■ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
□ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2 1

REVISIONS BY ______ DATE ______
FILE 1402-037
BY ______ DATE ___-70
CHECKED BY ______
946.7 (REV. 6-61)

BORING 10

Surface Elevation 85 ± feet
MSL Datum

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Graph Symbol	Letter Symbol	Description
								0.5", asphaltic concrete
								boulder at 1'
							MH	Red-brown clayey silt (very stiff to hard, fill)
29.1	87.7	28		▩				
					5			
18.1		28		▩	10			becoming mixed with weathered to fresh gravel and rock fragments
					15			Boulders mixed with sandy and gravelly silt (fill)
		12/5"		▩				
								Boring completed at 19.0 feet on 8-11-70
								No water encountered

LOG OF BORINGS

NOTES:

▩ -Depth at which undisturbed sample was taken
⊠ -Depth at which disturbed sample was taken
☐ -Depth at which sample was lost during extraction
I -Depth and length of core run
Driving energy - 300-lb weight dropping 30 inches

Dames & Moore
PLATE A-2J

REVISIONS BY DATE
FILE 4402-037
BY CHECKED BY DATE
946.7 (REV. 6-61)

BORING 11

SURFACE ELEVATION 85 ± FEET
MSL DATUM

Moisture Content in %	Dry Density in PCF	Blows/Ft. on Sampler	Core and % Recovery	Samples and/or Cores	Depth in Feet	Graph Symbol	Letter Symbol	Description
					0			6", ASPHALTIC CONCRETE AND CRUSHED ROCK BASE COURSE
							MH	DARK RED CLAYEY SILT (FIRM TO STIFF, FILL)
35.6	87.4	12		▩				
					5			
34.6	76.6	8		▩				
					10			FEW SMALL BOULDERS 9.5' TO 10.5'
34.6	85.4	4		▩				
					15			BECOMING MIXED WITH DECOMPOSED BASALT FRAGMENTS
38.6	80.8	12		▩				
							MH	GRAY CLAYEY SILT (STIFF TO HARD DECOMPOSED BASALT)
44.7	70.6	20		▩	20			

BORING COMPLETED AT 20.5 FEET ON 8-11-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2K

REVISIONS BY ____ DATE ____
FILE 4402-037
BY ____ DATE ____
CHECKED BY ____

946.7 (REV. 6-61)

BORING 12

SURFACE ELEVATION 104 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	GRAPH SYMBOL	LETTER SYMBOL	DESCRIPTION
								6", ASPHALTIC CONCRETE AND BASE COURSE
							MH	DARK RED CLAYEY SILT (STIFF OR IN LOOSE CHUNKS, FILL)
27.5	78.4	10		▩				
					5			
		6		▩				
					10			
							MH	RED AND GRAY CLAYEY SILT (VERY STIFF, DECOMPOSED BASALT)
50.4	64.6	10		▩				
					15			
50.5	67.5	14		▩				

BORING COMPLETED AT 19.5 FEET ON 8-12-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
⊠ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300 -LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2L

REVISIONS BY ____ DATE ____
FILE 4402-037
BY W Bum CHECKED BY DAC DATE 9-1-70
946-7 (REV. ...)

BORING 13

SURFACE ELEVATION 105 ± FEET
MSL DATUM

MOISTURE CONTENT IN %	DRY DENSITY IN PCF	BLOWS/FT. ON SAMPLER	CORE AND % RECOVERY	SAMPLES AND/OR CORES	DEPTH IN FEET	GRAPH SYMBOL	LETTER SYMBOL	DESCRIPTION
					0		MH	RED-BROWN CLAYEY SILT (HARD, FILL)
24.6	81.4	26		▩				
					5			
								BOULDER AT 8 FEET
32.0		14/6"		▩				GRADING TO FIRM WITH ROCK FRAGMENTS
					10			BOULDER AT 9.5 FEET
35.5	81.2	3		▩				GRADING TO SOFT
					15			
							MH	RED-BROWN CLAYEY SILT (HARD, DECOMPOSED BASALT)
51.5	64.7	16		▩				

BORING COMPLETED AT 18.5 FEET ON 8-12-70
NO WATER ENCOUNTERED

LOG OF BORINGS

NOTES:

▩ -DEPTH AT WHICH UNDISTURBED SAMPLE WAS TAKEN
☒ -DEPTH AT WHICH DISTURBED SAMPLE WAS TAKEN
☐ -DEPTH AT WHICH SAMPLE WAS LOST DURING EXTRACTION
I -DEPTH AND LENGTH OF CORE RUN
DRIVING ENERGY - 300-LB WEIGHT DROPPING 30 INCHES

DAMES & MOORE
PLATE A-2M

REVISIONS BY ___ DATE ___
FILE 4402-037
CHECKED BY DSR DATE ___ - 7
REV. 6-61)

DAMES & MOORE

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CABLE: DAMEMORE TELEX: 63-4100

February 22, 1971

Yasuo Arakaki, Consulting Engineer
914 Ala Moana Boulevard
Honolulu, Hawaii 96814

Gentlemen:

Recommendations regarding Footing Bearing Pressures
Proposed Rigid-Frame Culverts
Pearl City, Oahu, Hawaii
for City and County of Honolulu

In response to your request, we have reviewed soils data in our files and developed recommendations regarding soil bearing pressures which may be used for design of rigid-frame culverts for your Pearl City drainage project. The data originally were developed during an investigation of certain aspects of that project for you.*

At the time of our investigation, four new box culverts were planned as replacements for existing box culverts. Now we understand that rigid-frame "inverted-U" culverts may be used instead. For this rigid-frame type of culvert, allowable bearing pressures under the footings become more important.

Although our investigation was directed primarily toward evaluating lateral earth pressures on the proposed box culverts, our borings and samples extended to depths slightly below planned invert level. Therefore, some data were available for evaluating allowable bearing pressures.

Our borings adjacent to the existing culverts showed somewhat variable soil conditions near invert level. This is not surprising, since soils may have been partially disturbed or replaced during construction of the existing culverts. Also, alluvial soils along the drainage channel could be expected to be variable. However, in at least half of the borings, in-place decomposed basalt, having the engineering properties of a stiff clayey silt, was encountered at

*See our report "Subsurface Investigation, Proposed Drainage Tunnel and New Box Culverts, Pearl City, Oahu, Hawaii for City and County of Honolulu", dated September 2, 1970.

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elevations which ranged from about two feet above to three feet below invert level. We believe that this material is the best that would be available for support of culvert footings. We also believe that this soil may exist at reasonable depths below all the culvert areas.

If the culvert footings are constructed on in-place decomposed basalt, we recommend that they be proportioned for a maximum design bearing pressure of 4000 psf. Under this loading, we do not believe that settlement would be a problem, although settlement would be proportional to the size of the footings used. We should like to review potential settlement, as well as potential lateral loads on adjacent soils, after tentative designs of the culverts have been made.

Since our recommended bearing pressure is based on a specific type of soil under the footings, inspection of all footing excavations during construction is definitely recommended.

Yours very truly,

DAMES & MOORE

David C. Liu

David C. Liu

DCL DRR mw

(seven copies submitted)